Response of Conventional Steel Stud Wall Systems under Static and Dynamic Pressure

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Abstract: This research effort focuses on the evaluation of existing design standards for cold-formed steel stud walls and the development of retrofit wall systems. Full-scale wall systems are tested under uniform static pressure using a vacuum chamber. The resistance functions obtained are used to model the dynamic behavior of the walls and to predict performance under blast conditions. This paper focuses on defining the static resistance of nonload-bearing steel stud walls with slip track connections and their performance under external explosions. Simple modifications to existing design practice have significantly improved the blast performance of the steel stud walls. Maximum blast resistance is achieved by using steel angles connected to the studs and anchored to the floor and ceiling. The static and dynamic performances of five full-scale steel stud wall systems are presented in this paper.

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Introduction

In recent years, the need to quantify and understand the behavior of cold-formed steel stud walls under blast loading has become prevalent. As the popularity of cold-formed steel stud walls rises throughout the world and the material availability grows, the construction of such walls has also increased. To protect people inside structures using cold-formed steel stud walls from the threat of explosion, researchers have been working to develop blast design criteria for these walls. Although dynamic models for blast design have long been in existence, obtaining information on stud wall behaviors to be used in a dynamic model requires investigation.

Dynamic modeling first requires that the behavior of the structure under static loading be defined (Biggs 1964). For blast design of a stud wall, the static resistance function of a wall loaded with uniform pressure must be developed (Salim et al. 2003). To some degree, existing design methods may provide answers, but blast design requires information beyond that covered under previous studies for conventional design. Numerous works have taken in-depth looks at the behavior of steel studs under lateral out-of-plane loading for designs against wind loads; however, blast resistance views the "capacity" of a wall from a different perspective. The design of the stud wall for blast resistance may rely on the energy absorbed beyond the elastic region of the stud wall resistance. In other words, unlike design for wind loads, which attempts to keep the structure behaving elastically, blast design may welcome heavy damage to the structure, and in doing so absorb more blast energy. Once the static resistance function for a given stud wall is known, a single degree of freedom (SDOF) dynamic model can be used to predict the behavior under blast loading.

This paper provides information pertaining to the behavior of steel stud walls under uniform lateral pressure for use in dynamic analysis. The research focuses on defining how various details of construction affect the behavior of the wall systems. The details under consideration in this document pertain to nonload-bearing slip track stud walls; however, other systems are also being studied.

Through full-scale static testing of walls under uniform lateral pressure and component testing, the effects of stud bending behavior and connection details on the response of the wall systems will be defined. The data for experimental walls will provide the static resistance functions for steel stud walls, which will be used in an SDOF dynamic model to predict their behavior under a given blast loading (Biggs 1964).

In addition to defining the static resistance of steel stud walls, the scope of this research also includes providing simple modifications to construction techniques that will provide higher resistance. For example, adding a screw to a connection detail may provide a significant increase in resistance. The behaviors of the walls during test are used to propose changes to improve wall performance. These design alterations are geared toward increasing ductility or energy absorption, preventing brittle connection failure, and providing efficiency in design of stud walls.

This paper summarizes the initial testing performed on nonload-bearing walls with slip track connections. However, the walls tested vary slightly in connection details, thus providing an insight into the effects of these variations on the wall behavior. This paper focuses on the testing of four walls with slip track connections. These nonload-bearing walls, referred to as walls W1–W4, will provide information on conventionally constructed stud walls. The performance of a fifth wall, W5, with a properly

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designed anchorage system to allow the steel studs in the wall to develop their full strength and ductility, is also presented.

Background

Many years of extensive research on cold-formed steel technology has culminated in the *Cold-Formed Steel Design Manual* published by the American Iron and Steel Institute (AISI 1997). Evaluation of the static resistance functions of stud wall systems can be broken into two major topics found within the AISI specification. The first is the bending behavior of the steel stud members under uniform pressure, and connection details are the second. The bending of the beam and the behavior of the connections are inherently intertwined, as they are reliant on each other. Stability and connection limitation can also control the wall performance.

Bending Capacity of a Channel Section

The flexural rigidity for cold-formed beams often does not behave in accordance with simple beam theory. The stiffness of the beam can be substantially lower at high loads than at lower loads, and the predicted stresses using simple beam theory can be much less than that in a cold-formed section under a given load (Walker 1975). When considering the bending strength of a stud, the behavior of the compression flange and the portion of the web under compression can play an important role. AISI (1997) specifications define an effective section width. If it is found that the compression elements are not fully effective, and hence the moment of inertia is not based on the full section, the moment of inertia of the beam can vary along the span with the moment diagram. The results are a lowered yield of the beam and a higher deflection due to a lower moment of inertia.

The nominal moment of the section is equal to the effective yield moment, M_y, which is based on the effective section modulus, S_e, and the yield strength, F_y, as follows:

$$M_n = M_y = S_e F_y \tag{1}$$

The effective section modulus is based on effective areas of the flanges and web. These effective widths account for local buckling and postbuckling strength (Shan 1994). For the channel sections evaluated in this paper, the neutral axis will be at middepth or closer to the tension flange, depending on the effectiveness of the compression elements. In this case, the effectiveness of the web, compression flange, and the stiffener must be checked, and if they are not fully effective, effective widths should be found for computing the effective section modulus.

In addition to the moment of inertia, the effects of the shear center must be inspected. The shear center becomes an issue with channel sections because the asymmetry of the section will cause rotation. This will manifest itself through asymmetric behavior of a wall; however, with adequate bracing through sheathing, the effects can become negligible.

Limit States

In practice, the response of stud walls will be limited by several factors. Many construction details may limit the ability of a stud to reach the phases described earlier. Aside from connection failures, which play a major role in the behavior of a wall, the bending capacity of the wall can be limited by yield, flange buckling, lateral torsional buckling, and shear buckling.

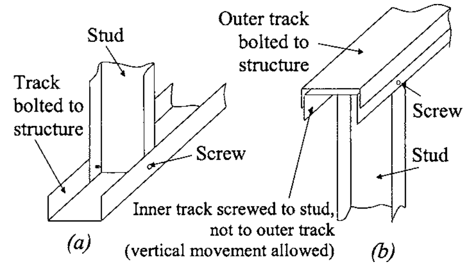

Fig. 1. Stud-to-track connections: (a) bottom track; (b) top slip track

The capacity of a steel stud is defined by the yielding of the beam flanges. This is generally the case for the design specifications used today. In Procedure I of the AISI specifications, the capacity is defined by yielding, which may or may not be considered conservative, depending on the boundary conditions. In blast resistant design, it is beneficial to consider capacity up to total failure of the wall. If the wall is pinned at the top and bottom, it will likely go beyond yield, but if the wall is modeled by pin-roller support configuration, such as a slip track (Fig. 1), the wall capacity will be limited by yield.

For long, unbraced thin-walled sections such as steel studs, lateral-torsional buckling (LTB) will often affect bending capacity (Put et al. 1999a,b; Hancock 1997). If the unbraced length becomes large, the section will have a tendency to move out-of-plane. However, adequate sheathing of stud walls may create sufficient bracing against LTB unless the sheathing spanning between two adjacent studs breaks. To account for LTB, AISI (1997) defines a critical LTB stress, F_c, at which the moment capacity of a beam occurs as

$$M_n = S_c F_c \tag{2}$$

Under uniform loading of a cold-form steel stud, it is unlikely that shear will control the design. However, the shear capacities are presented in AISI (1997).

Behavior of Stud-to-Track Connection

As mentioned earlier, behavior of a stud wall hinges on two items: (1) the capacity of the studs; and (2) the number and size of connections that will allow that capacity of the studs to be utilized. This section will focus on the latter. Existing material pertaining to connection details found in standard construction will be discussed. These details will include the behavior of the stud, the tracks, and the stud-track connections for both stud-to-track and stud-slip track details. When discussing the failure of these standard connections, the web crippling of the stud, the screw strength, and the resistance of the track to buckling and flange bending must be considered. A diagram of a screwed track and a slip track support is shown in Fig. 1.

One of the failure modes at a stud connection is web crippling. Web crippling, also known as bearing, is the failure of the web due to high stress intensity from a concentrated load or reaction (Hektrakul and Yu 1978; LaBoube 1997; Langan 1994). Buckling of flat rectangular plates under a locally distributed edge force has been studied in detail and is well defined. However, web crippling, as opposed to simple flat plate buckling, involves many complicated factors, including the following (Yu 2000):

1. Nonuniform stress distribution under the applied load and adjacent portions of the web;
2. Elastic and inelastic stability of the web element;

3. Local yielding in the immediate region of load application;
4. Bending produced by eccentric reaction;
5. Initial out-of-plane imperfection of plate elements; and
6. Interaction of the beam web and flange.

For these reasons, the AISI specifications have been based on extensive experimental evaluations by Winter and Pian (1946), Zetlin (1955), and Hetrakul and Yu (1978). However, these highly empirical prediction formulas do not directly model the connections found at a stud-to-track connection. Of the several scenarios given for web crippling in the specifications, those that most closely model connections found in stud walls will be discussed herein.

Due to the stud-to-track connection, web crippling becomes more complicated. The tracks are rather flexible, and screws connect the track through both flanges but not as two compressive forces. Although the model presented by AISI (1997) is not a perfect representation of the stud-to-track connection, the equation accompanying one-flange loading (AISI 1997) may provide the closest representation of experimental loading. However, its usefulness will depend on the screw and track behavior.

Another important limitation on the strength of a conventionally constructed stud wall is the capacity of the screws. The strength of a screw connection, as per AISI (1997) specifications, is divided into shear and tension failures. Shear is divided into connection shear and shear in the screws, and tension is divided into pull-out, pull-over, and tension in the screws. However, the complicated behavior of the stud-to-track connection yields various and complicated behaviors in the screw connections, and it may be the case that screw failure becomes critical after failure of another sort, such as web crippling, occurs. Additionally, quality control of screws and their installation introduces great variability into predictions.

Beyond the capacity of the stud and the screws, the wall relies heavily on the capacity of the tracks to which the studs are connected. Trestain (2001) states that the crippling capacity of a stud can be developed if the thickness of the track section is equal to or greater than the thickness of the stud. This means that current design for a stud-to-track connection gives a connection capacity equal to the web crippling capacity of a stud if the track is sufficient. Current design does not mention any prevention of web crippling of the stud from the stiffness of the track. Although it is not accounted for in conservative design standards, it is a possibility that the track "stiffens" the web of the stud through the screw connections.

The failure of a flexible slip track can easily govern the capacity of a stud wall. Work done by Trestain (2001) explores the failure of a flexible slip track as the inner slip track pushes over the flange of the outer track. This topic applies to Wall W1, which utilizes a flexible slip track. Walls W2–W4 are nonload-bearing but with rigid slip tracks made of structural steel that will not fail due to opening of the track. Wall W5 does not use a track, but rather a direct connection to the floor and ceiling.

Experimental Investigation

Due to the complex nature of testing infill walls subjected to uniform pressure, available experimental methods are limited. Restrictions in testing procedures can hinder accurate modeling of the response of the wall system and/or the loading scenario associated with the wall systems that are to be tested. For these reasons, the majority of this research was focused on full-scale testing in a static vacuum chamber. Additionally, work utilizing component testing can be found in Salim et al. (2003); however, component testing is not discussed in this paper. The static vacuum chamber used to test 3.05×3.66 m (10×12 ft) walls is discussed here.

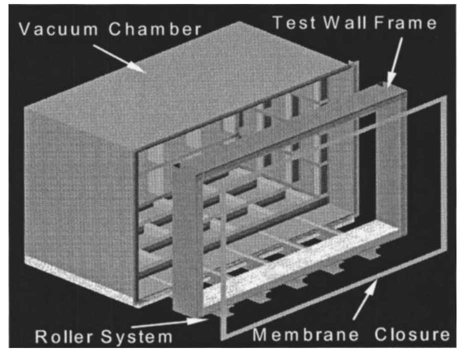

Fig. 2. Static vacuum chamber

Due to several inherent problems with existing experimental procedures, as discussed by Brown (2004), a new setup was devised and built for testing full-scale walls under uniform pressure. A static vacuum chamber, shown in Fig. 2, is used to apply a uniform pressure on the walls. The vacuum chamber is essentially a box with an open face where the test wall is built. The chamber consists of structural tubing covered by sheet steel and a removable frame in which the test specimen is built. With the wall built in this frame, a latex membrane is placed over the open face of the chamber and clamped down between a small frame and the chamber. A vacuum can then be created in the chamber by a vacuum pump hooked to the chamber, and the wall is loaded by the atmospheric pressure on the outside of the wall. Pressure and wall deflection can be recorded through data acquisition, and the full-scale static resistance is recorded.

Pressure data from three pressure transducers along the back of the chamber were recorded and used as an average for the resistance function. Inward deflections were acquired at five locations using string potentiometer displacement gauges (pots B1–B5), one located at the center, two at quarter heights, and two at quarter width. String pots B1–B5 were mounted on the back wall of the chamber with strings attached to the inside flanges of the corresponding studs.

Test Wall Descriptions and Static Test Results

Aside from the use of a rigid slip track for Walls W2–W4, the construction details used for these walls can readily be found in public and private construction. Information concerning standard construction can be utilized to rate the level of protection provided by existing structures. The data from these tests can be used to define the behavior of conventional walls under uniform lateral pressure.

At this point, it is important to consider the definition of the track components of the nonload-bearing type wall. Fig. 1 displays the connections used in a nonload-bearing wall. In subsequent discussion, the track used to connect the bottom of the wall will be referred to as the bottom track. For the top, the track connected to the stud will be called the inner slip track, and the encompassing track connected to the surrounding structure will be referred to as the outer slip track.

As mentioned earlier, Walls W2–W4 are all nonload-bearing designs. The term nonload-bearing implies that vertical loads that

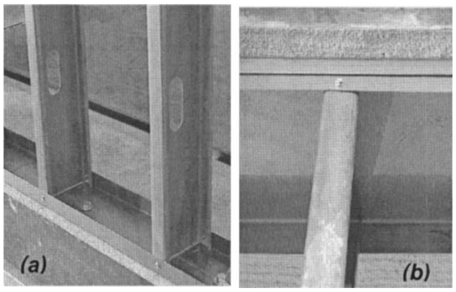

Fig. 3. Wall W1 tracks: (a) bottom; (b) top slip

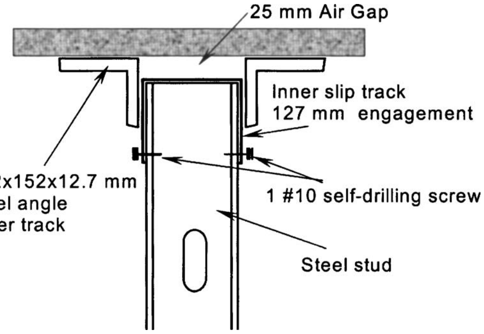

Fig. 4. Wall W2 rigid slip track detail

the surrounding structure is subjected to are not transferred into the stud wall. This is accomplished through use of a slip track (Fig. 1). A slip track connection is made up of a typical track-to-stud connection nestled inside a larger track, which is connected to the structure above. No mechanical connection between the outer and inner track exists, and a gap between the top of the inner track and the outer track allows deflection of the structure without axially loading the studs. Wall W1 is a typical example of such a wall. Walls W2–W4 represent the same wall type, but with a less typical slip track detail. These walls utilized a rigid outer slip track rather than the flexible track used in W1 to prevent premature failure resulting from the typical flexible outer slip track. Studs in Wall W5 connect directly to the top and bottom slabs through 12.7 mm (1/2 in.) steel angles.

In addition to being nonload-bearing, these test walls shared other details. All five walls were 3.05×3.66 m (10×12 ft) walls, created with 227 MPa (33 ksi), Clark 600S162-43 studs spaced at 406 mm (16 in.). Information on this section is provided in *Clark* (2000). Additionally, the bottom track and the inner slip track were Clark 600T250-43 track for all tests except W5. The specifics to each wall are given in this section.

The data recorded through full-scale vacuum chamber testing were used to develop the static resistance function of the walls. Resulting pressure-deflection plots for these tests provide an accurate account of how these walls behave under uniform lateral pressure. In this section, the static resistance for each wall is given along with a graph overlying the pressure versus center deflection response of all four walls. In addition to the static resistance functions of each wall, the performance of the walls to blast loading is presented.

Construction details and results for these walls, tested in the static vacuum chamber, are defined in this section. The dynamic performance of all walls under blast pressures and impulse is also presented. The field performance of Wall W5 under live explosion test is presented.

Wall W1—Description

Full-scale test Wall W1 was made up of the 16 gauge studs and tracks as described previously. However, the significance of this wall is found in the flexible slip track connection. The inner and outer slip tracks were both 16-gauge [1.37 mm (0.054 in.) thick], unlike W2–W4 tests, which utilized rigid outer tracks. A single #10 TEK screw was used to attach the studs to the track, one screw per flange (Fig. 3). Also, the sheathing consisted of 16 mm (5/8 in.) gypsum boards on the inside and outside of the wall. All sheathing was attached to the studs with #8 self-drilling drywall screws placed 0.305 m (1 ft) on center. The bottom track and the outer top track were bolted to the test frame with 22 mm diameter (7/8 in.) bolts with 57 mm (2 1/4 in.) outside-diameter washers. Anchors were placed at 64 mm (2 1/2 in.) offsets from the web of the studs. Additional details are provided by Muller (2002).

Wall W1—Test Results

In Fig. 4, the resistance of Wall W1 is given; however, unlike the following tests, only the deflection from the centermost string pot (B2) is given, because little difference could be seen in the other deflection readings. During testing, the gypsum board on the outside of the wall ruptured at a low pressure, which was immediately followed by lateral-torsional buckling (LTB) of stud #7 (Fig. 5) at approximately 4.8 kPa (0.7 psi). At slightly higher pressure, 5.5 kPa (0.8 psi), the flange of the flexible outer slip track, on the inside face of the wall, opened, allowing the top of the wall to move in freely. The failure mechanism of Wall W1 is shown in Fig. 5. After the pressure needed to fail the slip track was found, the decision was made to prevent such a failure in subsequent testing by using a rigid track and oriented strand board (OSB) for exterior sheathing.

Wall W2—Description

The objective of this test was to determine the resistance function and failure modes of a nonload-bearing wall with rigid slip track. With a rigid slip track, the failure of the slip track through bending of the track flange (as shown in the Wall W1 failure) is eliminated. Therefore, data for a nonload-bearing wall with a rigid slip track will go beyond the flexible slip track limit. In addition to the slip track change, the exterior sheathing only was changed to 16

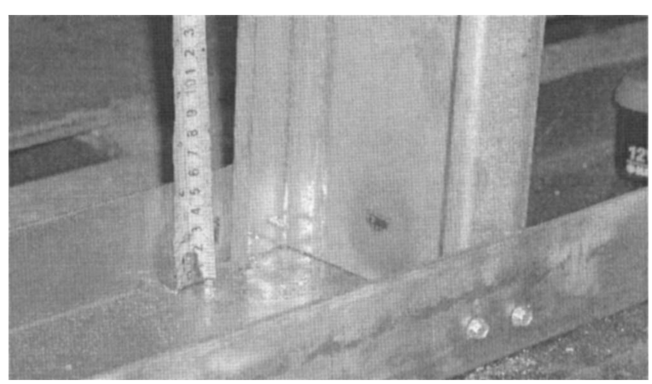

Fig. 5. Bottom stud-to-track connection using two #10 TEK screws for W3 and W4

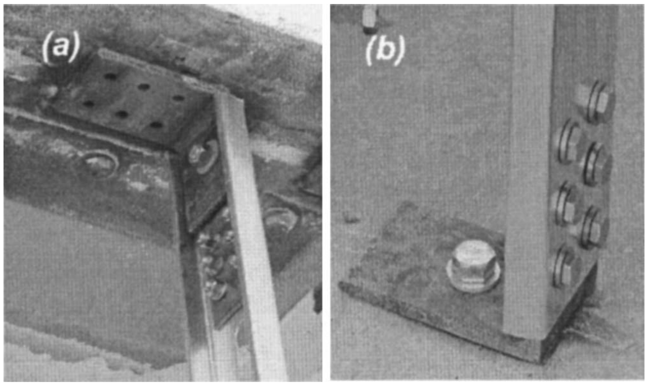

Fig. 6. Wall W5 stud-to-floor anchorage using 12.7 mm thick steel angle: (a) top; (b) bottom

mm (5/8 in.) oriented strand board (OSB), instead of gypsum board, to prevent premature sheathing failure and lateral instability of the studs.

The bottom track was bolted to the test frame the same way as in W1. The top track system, shown in Fig. 6, was a slip track system consisting of an outer 152×152×12.7 mm (6×6×1/2 in.), A248 (A36) steel angle on front and back simulating a rigid outer slip track. A 152×152×1.37 mm (6×6×0.054 in.) C-section was used for the inner slip track connected to the studs using one #10 TEK screw per flange. The slip track insured a 25.4 mm (1 in.) air gap at the top of the track with 127 mm (5 in.) of engagement length between the inner track and the outer rigid angles.

Wall W2—Test Results

Fig. 7 shows a linear response of the wall followed by a quick jump in deflection and finally a rise to a peak strength marked by catastrophic failure. The sudden jump in deflection, following the elastic response, represents a rapid displacement of the top connection. This movement resulted from the web crippling of the studs, shown in Fig. 8(b), due to the concentrated reactions. This observation is also confirmed by the shift of the above midheight deflection, B4, from being the least deflection measurement to the greatest. The failure of the bottom stud-to-track connection created a sudden failure arising from a combination of screw failure and web crippling.

Catastrophic failure of the wall was achieved at nearly 11.4 kPa (1.65 psi). Very little movement was observed leading up to the wall failure. After the wall came to rest, the tops of the studs remained attached, with the slip track pulled down about 10 in. on the left to almost none on the right (Fig. 8). The bottom ends of

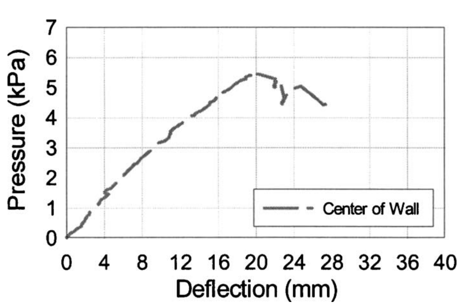

Fig. 7. Wall W1 static resistance

Fig. 8. Wall W1 posttest

the studs, however, were torn from the bottom track and were resting inside the chamber [Fig. 8(c)]. The bottoms of the studs were crippled for a length of approximately 76–152 mm (3–6 in.). Both track flanges were bent into the chamber. The screws connecting the stud to the track were pulled from the outer flange of the track and sheared on the inside flange [Fig. 8(d)]. The OSB was broken across the bottom of the wall as the bottom moved in. Although the failure of the studs was fairly consistent across the wall, the amount the slip track dropped was not symmetric.

Wall W3—Description

Continuing the goal of Wall W2, Wall W3 was designed similarly to W2, but with a few exceptions. As expected, the failure capacity of W2 was larger than W1 because of the rigid slip track; however, another sudden failure was reached when the studs ripped out of the bottom track. In light of the undesirable failure mechanism of W2, the studs of wall W3 were connected to the bottom track with an extra screw per flange. The construction details of wall W3 match the details of wall W2 with the following exceptions:

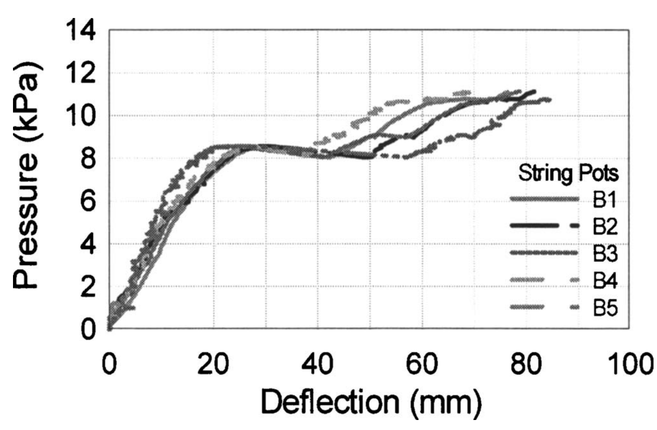

Fig. 9. Wall W2 static resistance

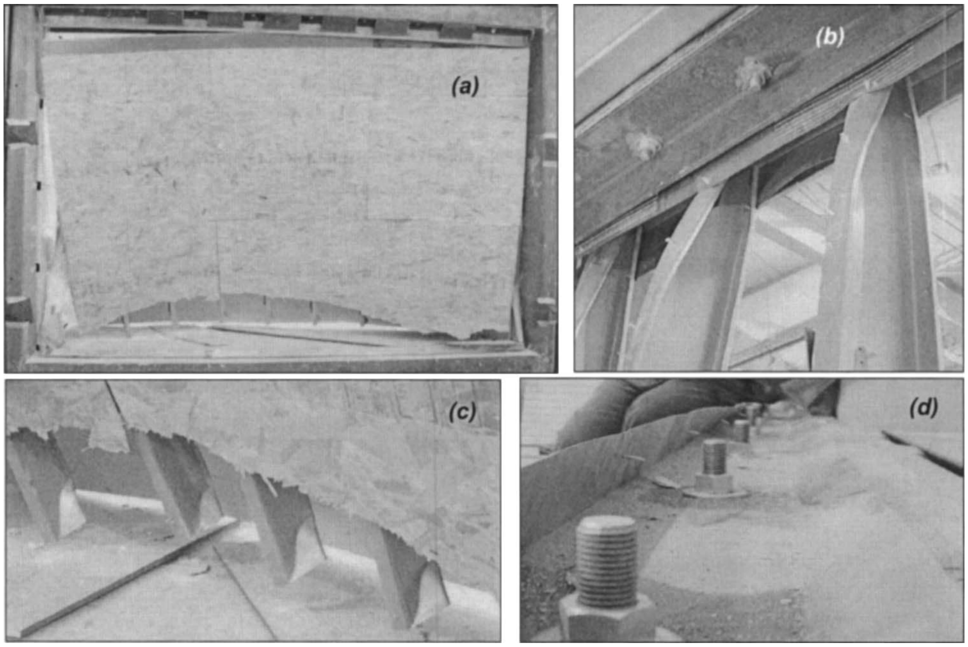

Fig. 10. Wall W2 posttest: (a) overall condition; (b) top connection web crippling; (c) screw failure at bottom; (d) flexible bottom track opening

1. Two #10 TEK screws were used per flange per stud to connect the studs to the bottom track (Fig. 9).
2. $102 \times 102 \times 12.7$ mm($4 \times 4 \times 1/2$ in.) angles were used for the outer rigid slip track. This lowered the engagement from 125 mm (5 in.) to 76 mm (3 in.).
3. The wall was "flipped" horizontally; i.e., all stud openings pointed right in Wall W2, but were installed to point to the left, and the OSB was installed in mirror fashion.

Wall W3—Test Results

As Fig. 10 shows, Wall W3 failed in a very different manner from W2. However, behavior up to the peak load was similar. The static response shows that just under 13.8 kPa (2.0 psi) the wall experienced a jump in deflection due to web crippling at the top connection, similar to W2. The graph shows that the deflection B4, which is located at the 1/4-point from the top of the wall, goes from the smallest deflection to the greatest during this phase. After the onset of web crippling, the wall continued to deflect, but with a slightly lower stiffness. Upon reaching about 9.65 kPa (1.40 psi), the wall formed "hinges" at the OSB joint at 0.304 m (1 ft) below midheight (Fig. 11). When this yielding occurred, the bottom track bent and the slip track came down on the right and remained jammed on the left. This is noted on the graph by a large loss in pressure and gain in deflection. In Fig. 10, the downward linear portion just after maximum pressure does not include data because the test was not deflection controlled. Following the rapid deflection and loss of load, the wall began to take little additional load, which could be attributed to the screws at the bottom and friction in the rigid slip track at the top.

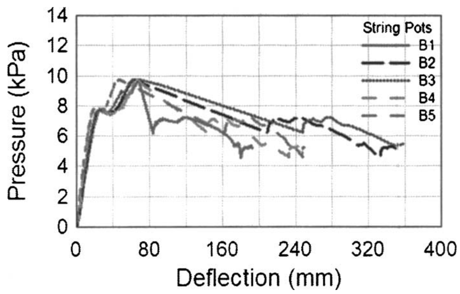

Fig. 11. Wall W3 static resistance

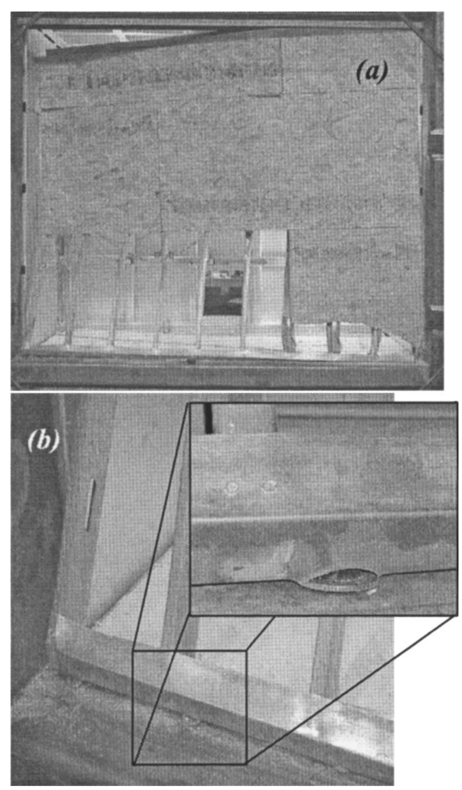

Fig. 12. Wall W3 posttest: (a) overall showing yield near midheight; (b) bottom track rotation showing failure around washers

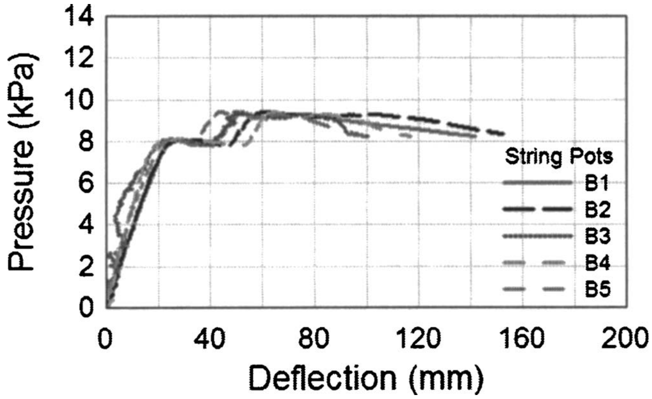

Fig. 13. Wall W4 static resistance

Wall W4—Description

Wall W4 was performed as a verification of W3, and its construction matched the details of W3 except that the studs faced the opposite direction. The flanges were pointing to the right as in W2. This change was made to verify the asymmetric failures seen for all the nonload-bearing wall tests.

Wall W4—Test Results

Wall W4 resulted in a viable confirmation of Wall W3. The stages described in the previous section for W3 are nearly identical to this test. The overall behavior was closely comparable. The only notable difference arose after peak pressure. In this case, the deflection was not as large and the pressure did not drop significantly. The load-deflection response is presented in Fig. 12, which gives data points during the horizontal softening part of the resistance of the wall. Unlike the resistance of W3, which exhibited a "jump" in the data points after yield-buckling, wall W4 transitioned relatively softly during the postyield-buckling point. It is also seen that in Fig. 13 the behavior was also asymmetric but in the opposite fashion. This again was a function of the direction the studs faced, which made the wall twist in one direction or another.

The resistance functions of Walls W1–W4 are compared in Fig. 14, which shows the variation in energy-absorption capability of each wall represented by the area under the pressure-deflection diagram.

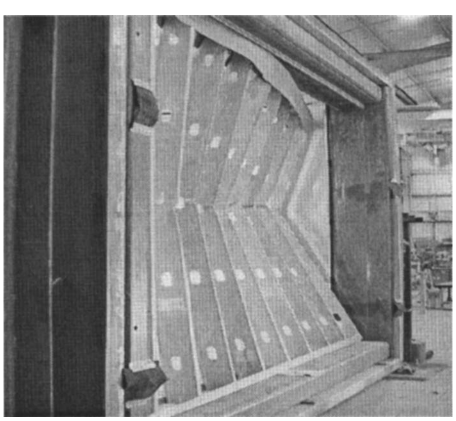

Fig. 14. Wall W4 posttest

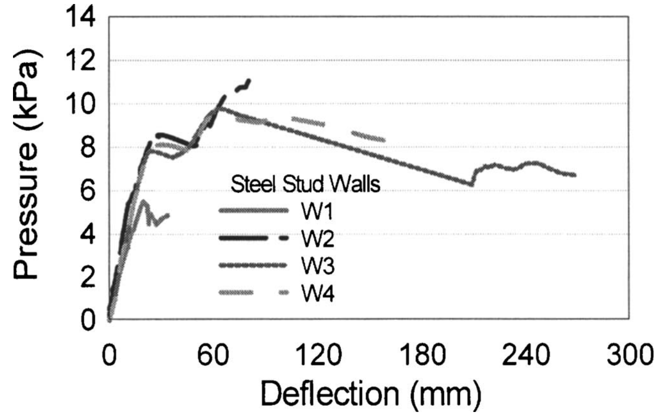

Fig. 15. Static resistance comparison for W1–4

Wall W5—Description

Wall W5 represents one of the first generation of blast resistant steel stud wall designs. The studs were connected directly to the floor slabs, without using any tracks. 12.7 mm (1/2 in.) steel angles and 24 mm (15/16 in.) anchor bolts were used to connect the studs to the top and bottom floors (Fig. 15). Details of Wall W5 are given by Muller (2002), Roth (2002), Dinan (2004), and Salim et al. (2003). 1.37 mm (0.054 in.) steel sheathing was used on the exterior of W5, and no sheathing was used on the interior. Instead, steel straps were used for lateral stability. The connection detail and sheathing used in W5 are intended to allow the studs to utilize their full strength and ductility for maximum blast protection.

Wall W5—Test Results

The maximum response of Walls W2–W4 was controlled by premature failures such as sheathing failure or by connection failure. Therefore, Wall W5 was designed to prevent all possible premature failure to ensure that the studs failed due to excessive elongation. If the studs fail in the cross section, the maximum possible ductility and energy-absorption capability of the wall will be achieved, a property needed for blast resistance (Salim et al. 2004; Salim and Townsend 2004). The posttest of W5 is provided herein; Fig. 16 shows the failure of the stud at midheight.

The resistance functions of the conventional walls (W1, W3,

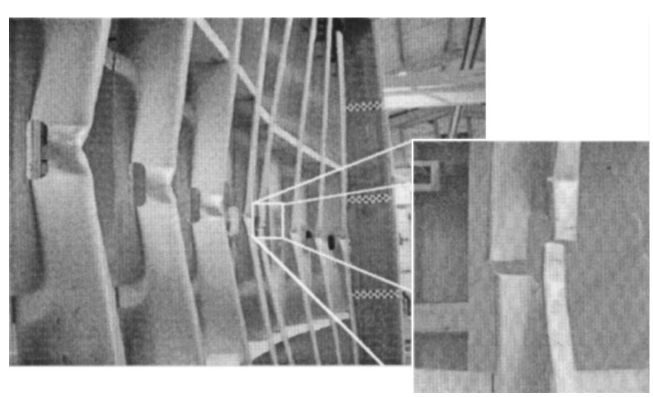

Fig. 16. Wall W5 after test—studs developed full capacity signified by rupture of stud at middle section

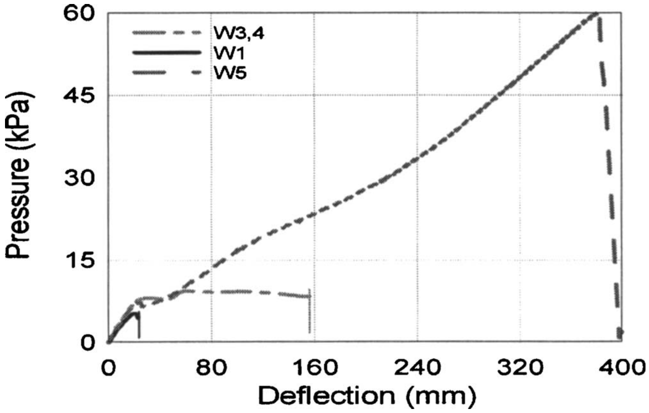

Fig. 17. Static resistance comparison for Walls W1, 3, 4, and 5

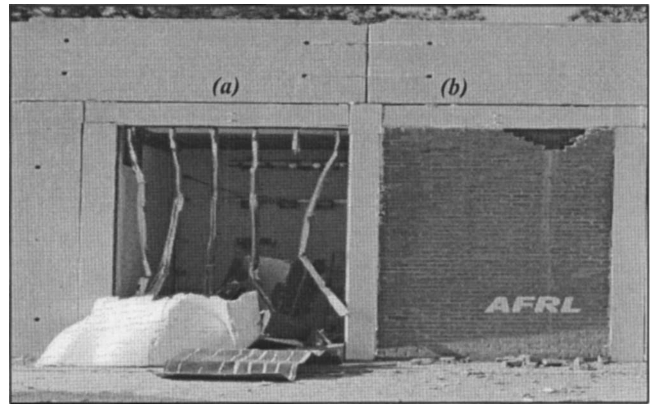

Fig. 18. Posttest exterior views of BREW-1 (similar to W5): (a) EIFS façade; (b) brick façade

and W4) are compared with that of the blast resistant wall (W5). The results are shown in Fig. 17, which demonstrates the blast resistance capabilities of steel stud walls.

Dynamic Evaluation of Walls under Simulated Blast Pressure

The resistance function of each wall was used with an SDOF numerical integration program, SSWAC (*Steel* 2003) to analyze their dynamic response to blast pressure. Details of the analytical model and engineering design methodology used are described by Salim et al. (2004). To quantify the level of blast resistance that a conventional steel stud wall system exhibits, the peak reflected pressure and impulse for an explosive charge weight and standoff distance were determined using TM 5-1300 (U.S. Department of the Army 1990). The peak pressure created by the explosion is 238 kPa (34.5 psi) and the peak impulse produced is 720 kPa-ms (104.5 psi-ms).

The blast performance of the walls was conducted using SSWAC (*Steel* 2003) with an external veneer wall using 192 mm wide × 55.5 mm tall × 89 mm deep (7 9/16 × 2 3/16 × 3 1/2 in.) clay bricks. The brick veneer provided mass, which contributes to the blast resistance of the walls.

Using the resistance functions given in Figs. 14 and 17, the dynamic responses of the walls under blast pressure are given in Table 1. The results show that Walls W1 and W2 will not survive the blast, whereas Walls W3, W4, and W5 will. Walls W3 and W4 are predicted to survive with almost no factor of safety, whereas W5 is predicted to survive with additional reserved capacity.

The blast performance of the walls was also predicted using a peak pressure of 228 kPa (33 psi) and a peak impulse of 1,378 kPa-ms (200 psi-ms). All walls were predicted to fail except Wall W5. Subsequently, Wall W5 was selected for field verification using live explosives.

Full-Scale Dynamic Verification

As part of the Blast Response of Exterior Walls (BREW) research program, a full-scale blast experiment (BREW-1) was conducted by the Air Force Research Laboratory at Tyndall Air Force Base. The purpose of the test was to validate the performance of the anchor systems in developing the full tensile capacity of the studs, to demonstrate the contribution of the mass to the wall response, and to compare the results of the experimental data with the prediction model (Salim et al. 2004).

Two steel stud walls with blast-design connections similar to W5 were tested. The walls were approximately 3.66 m (144 in.) tall and were attached at the bottom to a reinforced concrete slab using concrete anchors, and at the top to a steel plate (representing either a steel beam or an embedded steel plate in concrete) using a steel angle welded to the plate and a hole in the vertical leg of the angle to allow for a hinged connection (Fig. 15). One wall contained a brick façade with an area density of 1.46 kN/m^2 (30.5 lb/ft^2). The façade of the other steel stud wall consisted of a typical External Insulation and Finish System (EIFS) exterior with an area density of approximately 0.072 kN/m^2 (1.5 lb/ft^2). The exterior side of the studs was

Table 1. Blast Performance of Walls

	Static performance			Dynamic performance due to blast		
Wall test	Maximum deflection (mm)	Maximum energy (kPa-m)	Energy ratio[a]	Maximum deflection (mm)	Time at maximum response (s)	Deflection ratio[b]
W1	19.8	0.054	1.00	252	0.120	12.72
W2	76.2	0.581	10.8	129	0.055	1.69
W3&4	165.1	1.278	23.7	161	0.076	0.98
W5	337.6	6.041	111.9	185	0.071	0.55

[a]Computed as ratio of energy absorbed in Wall W1.

[b]Deflection ratio = (maximum deflection of dynamic performance)/(maximum deflection of state performance). A ratio greater than 1.00 means wall will fail.

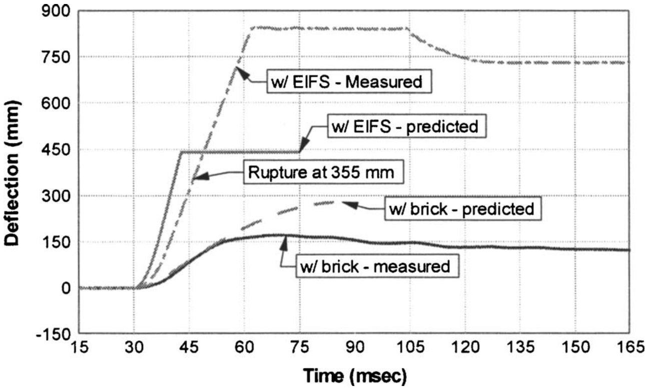

Fig. 19. Measured and predicted deflections at center of steel stud walls for field tests BREW-1

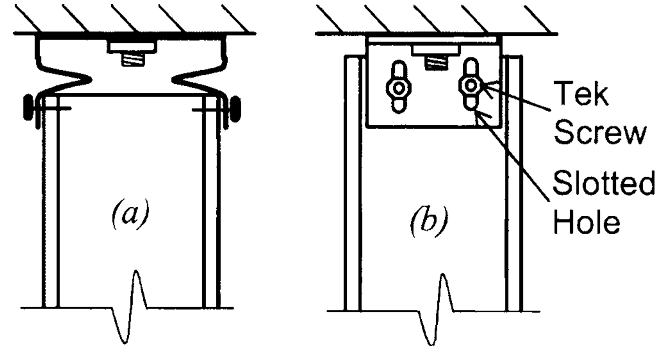

Fig. 20. Anchored steel stud top: (a) accordion track; (b) slip-clip

sheathed with 1.37 mm (16-gauge) sheet steel, and the interior studs were sheathed with a product consisting of 6 mm (1/4 in.) gypsum board glued to 1 mm (20-gauge) steel sheets to provide a finished interior surface while preventing secondary fragmentation from the gypsum board.

The walls were subjected to a live blast loading, and the post-test photo shown in Fig. 18 and the deflection measurements shown in Fig. 19 demonstrate the dramatic difference in wall response resulting from the inertial effects of the mass of the wall. Further information about this and similar dynamic field tests can be found in Salim et al. (2003), DiPaolo et al. (2003), and Dinan et al. (2003).

Conclusions and Recommendations

The static resistance functions recorded for the nonload-bearing walls discussed in this paper provide good resources for improving the knowledge for blast design. It was demonstrated that connections are a crucial element in blast design. Small changes made in the connections, i.e., changing flexible slip track to rigid or adding two screws per stud, greatly improved the ductility and therefore the energy absorbing capacity of the wall systems. In addition, the type of external sheathing is important in maintaining the stability of the wall system when subjected to blast pressures.

The performance of a conventional nonload-bearing steel stud wall under blast loading was improved by replacing the flexible track with a rigid track and the gypsum board with OSB. Additional improvement to the wall was achieved by using two screws instead of one to attach the studs to the bottom track. Additional improvement to the blast performance of the wall cannot be achieved without introducing "major" changes to the connections and anchorage design of the steel studs to the floor and ceiling slab.

Strong connections at top and bottom allowed the studs to utilize their full strength and ductility, which resulted in a significant capacity beyond yield. The adequate anchorage system proved that if connections are designed to allow for a tension membrane mechanism to develop after yield at midspan, the energy absorption of the system can increase drastically. The capacity available beyond yield-buckling, which is provided by the axial-deformation resistance of the stud, is generally limited by the capacity of the connections.

The nonload-bearing tests revealed that resistance of conventional wall designs is insufficient, but improvements in connections can increase capacity significantly. Wall W1, designed with a flexible slip track, failed at a very low pressure, but when the flexible slip track was replaced with a rigid outer track, the performance was greatly improved. Additionally, when two screws were added to the stud-to-track connection, ductility of the system was vastly increased and catastrophic failure was avoided. This indicates that simple measures can be made to add significant bending resistance of walls. Also, it is recommended that blast resistant walls should be considered with a connection other than a slip track. If the wall can be designed as load bearing, the resistance of the wall will greatly improve. For the design of nonload-bearing walls, a "slipping" connection used should be given much consideration. By connecting a wall with vertical movement restricted at the supports, for example, using an accordion track or a slip-clip (Fig. 20), some of the postyield strength of the studs can be utilized, more ductility may be achieved, and hence, more energy may be absorbed.

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